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DEVELOPMENT OF DELAY MODEL FOR NON-LANE BASED TRAFFIC AT SIGNALIZED INTERSECTION

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ABSTRACT

Delay is an important parameter for the measurement of the level of performance of signalized intersection. Well-defined procedures are available to measure delay for developed countries. Highway Capacity Manual (HCM) is widely used for capacity analysis of signalized intersection in North America and other developed countries. HCM and all other delay models have been developed assuming lane disciplined and more or less uniform traffic. Their applicability to non-lane based traffic conditions needs to be checked. In this study, a number of intersections have been selected in the city of Dhaka, Bangladesh, and field measurement of delay has been done by the method suggested in HCM 2000. The delay at each intersection is also estimated by Webster delay model, TRANSYT model, Akcelik's model, Reilly's model and Highway Capacity Manual (HCM) 2000 model. Based on regression analysis, modified delay model is developed for non-lane based traffic conditions prevailing in Dhaka city. Based on the findings of this study, it is recommended that the theoretical uniform delay (due to uniform arrival) and the incremental delay (due to random arrival and over saturated queues) in HCM 2000 delay model should be decreased by 20% and 85 %, respectively to better reflect field conditions where traffic does not follow any lane discipline. In addition to those, an intercept term has been proposed to use in the modified HCM 2000 model.

KEYWORDS: Acceleration, Deceleration Delay, Control Delay, Incremental Delay, Non-Lane Based Traffic, Uniform Delay

INTRODUCTION

Vehicle delay is perhaps the most important parameter used by transportation professional to evaluate the performance of signalized intersections. The importance of vehicle delay is reflected in the use of this parameter in both design and evaluation practices. For example, delay minimization is frequently used as a primary optimization criterion when determining the operating parameters of traffic signals at isolated and coordinated intersections. The Highway Capacity Manual (HCM) further uses the average control delay incurred by vehicle at intersection approaches as a base for determining the level of service provided by the traffic signals located at the downstream end of these approaches (TRB, 1997).

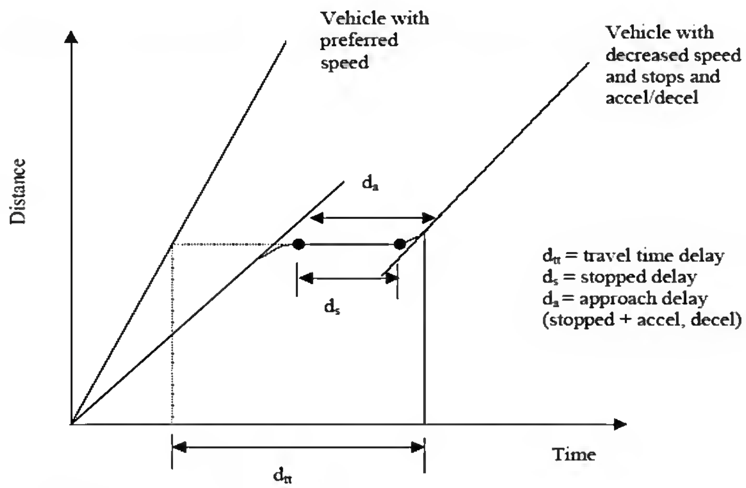
The popularity of delay as an optimization and evaluation criterion is attributed to its direct relation to what

motorists experience while attempting to cross the intersection. However, delay is also a parameter that is not easily determined. Teply (1989), for instance, has indicated that a perfect match between field-measured delay and analytical formula could not be expected. The difficulty in estimating vehicle delay at signalized intersections is also demonstrated by the variety of delay models for signalized intersections that have been proposed over the years.

Beside this, there are no proper guidelines available to estimate delay for non-lane based traffic conditions. In case of non-lane based traffic condition, lane markings, if present, are typically not followed by traffic. Traffic does not move in single file and there is a significant amount of lateral movement, primarily by the smaller-sized motor vehicles (motorcycles, mopeds, and scooters). Traffic movement at an intersection for lane based and non-lane based condition have been presented in Figure 1.

Most of the delay models developed before have assumed disciplined and more or less uniform traffic. Their applicability to non-lane based traffic conditions needs to be checked. Effect of lack of lane discipline on delay analysis needs to be considered. To achieve this goal, this paper first reviews the background material on vehicle delay estimation at signalized intersection for both lane based and non-lane based traffic condition. Then it gives insight into the field measurement of delay, theoretical estimation of delay, and calibration of HCM 2000 theoretical delay model for non-lane based traffic condition.

Figure 1: Traffic Movement at Signalized Intersection: Lane Based (Left) and Non-Lane Based (Right)



Source: McShane and Roess (1990)

Figure 2: Delays at Signalized Intersection

DELAY AT SIGNALIZED INTERSECTION

Delay at signalized intersections is computed as the difference between the travel time that is actually experience by a vehicle while going across the intersection and the travel time this vehicle would have experienced in the absence of traffic signal control. The diagram in Figure 2 further indicates that the total delay experienced by a vehicle can be categorized into deceleration delay, stopped delay and acceleration delay. Typically, transportation professionals define stopped delay as the delay incurred when a vehicle is fully immobilized, while the delay incurred by a decelerating or accelerating vehicle is categorized as deceleration and acceleration delay, respectively. In some cases, stopped delay may also include the delay incurred while moving at an extremely low speed. For example, the 1995 *Canadian Capacity Guide for Signalized Intersection* defines stopped delay incurred while moving at a speed that is less than the average speed of pedestrian (1.2 m/s) (ITE, 1995).

LITERATURE REVIEW ON DELAY ESTIMATION

A number of studies have dealt with the estimation of delay at signalized intersection, and most of them work for lane based traffic condition. Hurdle (1984) presented a paper to serve as a primer for traffic engineers who are familiar with capacity estimation techniques but have not made much use of delay equations. However, the model can be expected to give really consistent and accurate results. To obtain such results, one would need not just better models but better information about traffic patterns. Later, Lin (1989) evaluated the reliability of the HCM 1984 procedure based on field collected data, and discussed required modifications. Stopped delay was measured for single lane movements at seven intersections. To compare the HCM estimates with observed delays, the cycle lengths, green durations, yellow durations and saturation flow rates were also recorded using video cameras with built-in stopwatches. The evaluation reveals that the procedure tends to overestimate stopped delay at reasonably well-timed signal operations. Braun and Ivan (1996) also studied the methods for determining the average stopped delay by vehicles at eight signalized intersections during afternoon peak hour. They used the equations described in the 1994 version of the HCM, and found that the intersection approach delay estimated by the 1994 HCM was better than 1985 HCM. Later, Teply (1989) examined two approaches for measuring delay- a time-space diagram and a queuing diagram, and explained various problems related to each. The author concluded that, while delay cannot be measured precisely, it could be a useful engineering tool if it was calculated properly. In the same year, Hagen and Courage (1989) compared 1985 HCM delay computations with those performed by Signal Operation Analysis Package (SOAP) and by TRANSYT–7F Release 5. They studied the effect of degree of saturation, the peak hour factor and the period length on delay computations and on the treatment of left turns opposed by oncoming traffic. All of the models agreed closely at volume levels below the saturation point. But when condition became over saturated, the model diverged. Dowling (1994) tested the effect on accuracy of replacing most of the required field input data with the default values recommended in the HCM. The 1997 update of the *Highway Capacity Manual* (HCM) changed the concept of delay for level-of-service determination from stop delay to control delay. Powel (1998) suggested a rational and reasonable way to survey delay in the field, and then to translate this into total delay.

All of the previous delay estimation studies focused mainly on the lane based traffic flow, and they were developed assuming disciplined and more or less uniform traffic. However, very few studies have tried to investigate and then calibrate the delay models for non-lane based traffic condition. Hossain and McDonald (1998) developed a computer aided micro-simulation model to simulate the traffic operations in urban networks/corridors of developing countries. They used video and manual data obtained from Dhaka, Bangladesh for the purpose of calibration and validation of the

model. Although this study was not directly related to the delay estimation for non-lane based traffic condition, to the authors' knowledge it was the first of its kind to deal with non-lane based traffic flow. Later, Mathew and Radhakrishnan (2010) proposed a methodology to represent the non-lane based driving behavior and calibrated a micro simulation model for highly heterogeneous traffic at signalized intersection.

Hoque and Imran (2007) modified the Webster's delay model to make it usable under non-lane based mixed road traffic condition. They collected data using video camera at different signalized intersections of Dhaka city in Bangladesh, and measured the average delay per vehicle at each signal cycle. Based on these data, a model in the form of multiple linear regression was developed, which retained the first and second terms of Webster's delay formula but a modified adjustment term. The model has been calibrated to form a 'Modified Webster's Delay Formula', which was subsequently validated by comparing the expected delays with observed delays. The model provided a coefficient of correlation of 0.68, and all the independent variables were found to be statistically significant. The agreement between expected and observed delays was found to be satisfactory. The developed 'Modified Webster's Delay Formula' is only applicable for undersaturated traffic conditions. However, further research should be performed to develop modified delay models that can be applicable for both of the undersaturated and oversaturated non-lane based traffic conditions. The motivation of this study lies here.

STUDY AREA

For the present study five intersections have been selected in the city of Dhaka, Bangladesh. The study site is shown in Figure 3. All of the studied intersections are controlled by pre-timed signals. The intersections are selected considering their importance on the major arterial corridors in Dhaka city. Availability of nearby high-rise building also plays a role to select the studied intersections as the video cameras were placed on the roof top of those buildings. Those cameras capture the full queue length of traffic at the intersections.

Figure 3: Map of Dhaka City Showing Study Intersections

STUDY METHODOLOGY

Field measurement of delay has been conducted following the method suggested in HCM 2000, which is based on direct observation of vehicle in queue counts at the intersection. The method does not directly measure delay during deceleration and acceleration. Time-in-queue is measured by counting number of vehicles in queue at regular interval

of 10 to 20 seconds. Acceleration-deceleration delay has been estimated using the correction factor showed in Exhibit A16-2 of HCM 2000, and then added to time-in-queue delay to get the control (field) delay.

The delay that a particular vehicle experiences when it travels through signalized intersection approach depends on a number of factors such as arrival flow rate and distribution, signal timings, etc. In a real application environment, many of these factors are random variables which make the accurate prediction of delay a very complicated process. From the literature, a number of theoretical models have been found for delay estimation. However, in this study, five theoretical models have been used based on the availability of the model parameters, and their robustness for delay estimation. The following subsections briefly described the adopted theoretical models.

Webster Delay Model

Using deterministic queuing analysis, Webster (1958) developed a model for estimating the delay incurred by motorists at under saturated signalized intersection that becomes the basis for all of the subsequent delay models. The assumptions in this model are: 1) the rates are constant for the analysis period; 2) the demand is less than capacity; 3) the relation of delay to the pattern is deterministic; and 4) the arrival pattern of vehicles follows Poisson distribution. The developed model is:

$$d = \frac{C[1-(g/C)]^2}{2[1-v/s]} + \frac{(v/c)^2}{2v[1-(v/c)]} - 0.65(c/v^2)^{1/3}(v/c)^{2+5(g/c)} \tag{1}$$

Where,

d	=	Average overall delay per vehicle (sec/veh)
C	=	Cycle length (sec)
g	=	Effective green time for the approach (sec)
c	=	Capacity of intersection for the approach (veh/hr)
v	=	Actual or projected demand flow rate for the approach (veh/hr)
s	=	Saturation flow rate for the approach (veh/hr)
g/C	=	Effective green ratio for the approach
v/c	=	Degree of saturation.

However, it should be noted that Webster delay model is not applicable when demand exceeds capacity.

TRANSYT Model

TRANSYT model was developed to overcome the inherent limitations associated with Webster delay model in terms of oversaturated traffic state. While the exact TRANSYT model is quite complex, an approximation is proposed by Robertson's platoon-dispersion model (1979). The model is:

$$OD = \frac{15T}{c}\left[(v-c)+\sqrt{(v-c)^2+\frac{240v}{T}}\right] \tag{2}$$

Where,

OD = Overflow delay (sec/veh)

T = Analysis period (minutes).

The total delay (*d*) can be obtained by adding *OD* to uniform delay (*UD*). The *UD* can be estimated by the following equation:

$$UD = \frac{C[1-(\frac{g}{C})]^2}{2[1-\frac{v}{s}]} \tag{3}$$

Akcelik's Model

This model was developed by Akcelik for the Australian intersection analysis procedure. Akcelik (1980) assumed that total delay includes both acceleration and deceleration delays. The model is:

$$OD = \frac{T}{4}\left\{[(v/c)-1] + \sqrt{[(v/c)-1]^2 + \frac{12[(v/c)-(v_o/c)]}{cT}}\right\} \tag{4}$$

Where,

$$v_0 / c = 0.67 + s(g/600)$$

Equation 4 is valid for the cases in which $v/c > v_o/c$, otherwise the value of *OD* is equal to zero.

Reilly's Model

In preparing model for the HCM 1985 *Highway Capacity Manual*, Reilly and Gardner (1977) conducted extensive field studies to measure delays. They found that Akcelik equation consistently overestimated field-measured values, and recommended that the theoretical results be reduced by 50% to better reflect field conditions. The resulting equation is:

$$OD = 450\left\{[(v/c)-1] + \sqrt{[(v/c)-1]^2 + \frac{12[(v/c)-(v_o/c)]}{cT}}\right\} \tag{5}$$

Highway Capacity Manual (HCM) 2000 Model

After the release of HCM 1994, numerous researches have been undertaken to assess the changes that were made in the delay estimated model with respect to 1985 version of the model. Fambro and Rouphail (1997) proposed the delay that corrected some of the problems found in the 1994 HCM model and that is now the delay model found in the HCM 2000. In the HCM 2000, average delay per vehicle for a lane group is given by the following equation:

$$d = d_1 PF + d_2 + d_3 \tag{6}$$

Given that,

$$d_1 = 0.5C \frac{(1-\frac{g}{C})^2}{(1-Min(1,X)\frac{g}{C})} \tag{6.1}$$

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right] \tag{6.2}$$

$$PF = \frac{(1-P)f_P}{1-\frac{g}{C}}$$

Where,

d = Control delay (sec/veh)

d_1 = Uniform delay (sec/veh)

d_2 = Incremental or random delay (sec/veh)

d_3 = Residual demand delay to account for over saturation queues that may have existed before the analysis period (sec/veh)

PF = Adjustment factor for the effect of the quality of progression in coordinated system

k = Incremental delay factor dependent on signal controller setting (0.50 for pre-timed signals; vary from 0.04 to 0.50 for actuated controllers)

I = Upstream filtering/metering adjustment factor (1.0 for an isolated intersection)

T = Analysis period (hours).

DATA COLLECTION

Digital video camera was mounted at the roof of the building located near the intersection and was focused covering the one leg of the intersection. Care was taken to cover full queue formed on the study intersection approach. The recording was done for about 90 to 120 minutes during peak traffic conditions. Among the five intersections, four were recorded at 12:00 PM to 2.00 PM and another one at 5.00 PM to 6:30 PM. From the video record, vehicle counts were obtained. Data on signal timing, i.e. cycle length, number of phases, phase length were collected manually using stopwatch.

FIELD MEASUREMENT OF SATURATION FLOW

The average headway method based on time headway of departing vehicles cannot be used to measure saturation flow for non-lane based traffic condition. Because in non-lane based traffic flow, headways are difficult to observe, as vehicles do not move in lanes. Traffic is analyzed on the basis of total width of approach and hence, the option of vehicle counting is adopted. Saturation flow is calculated independently for each observed saturation period, and then averaged over observed cycles. Following the Road Note 34 (1963), it has been considered that the saturation period begins when

the green has been displayed for 3 seconds, and saturation period ends when the rear axle of the last vehicle from a queue crosses the stop line. All counted vehicles are added and the sum is divided by saturation period to get saturation flow in vehicles per hour. Description of study intersection approach with observed saturation flow in vehicles per hour at the eight lane groups has been presented in Table 1.

Table 1: Description of Study Intersection Approach with Observed Saturation Flow

Intersection	Approach Width (m)	Cycle Time (Sec)	Green Time (Sec)	No of Phase	Observed Saturation Flow (veh/hr)
New Market					
South approach	11.57	219	32	4	2463
North approach	11.9	219	47		3575
Bangla Motor					
South approach	9.23	135	100	2	4574
North approach	10.6	135	100		5568
Panthapath				3	
East approach	12.97	190	47		4734
Science Lab					
North approach	6.78	167	107	3	3029
East approach	7.8	167	47		3413
Sheraton					
East approach	12.68	158	68	3	5257

FIELD DELAY MEASUREMENT

Field measurement of delay was conducted at the five selected intersections. Traffic recording was captured at the selected intersection approaches covering the whole queue. HCM 2000 procedure was followed to calculate control (field) delay. Method suggested by the HCM (2000) is based on direct observation of vehicles-in-queue counts at the intersection. This method does not directly measure delay during deceleration and during part of acceleration, which are very difficult to measure without sophisticated tracking instrument. However, this method has been shown to yield a reasonable estimate of control delay. The method includes an adjustment for error which may occur when this type of sampling technique is used. It also includes another correction factor for acceleration-deceleration delay. In this study, the survey period began at the start of the red phase of the study approach, ideally when there was no cycle failure (no overflow queue) from the previous green period.

Recorded cassettes were replayed to retrieve data for delay calculations. The moment signal turned to red, cassette was paused and video cassette player (VCP) timer was set to zero. The overflow queue was excluded from queue counts. The reason for this exclusion results from the need for consistency with the analytical delay equation, which is based on delay to vehicles that arrive during the survey period. This time period might differ from analysis period which was typically considered 15 minutes as per HCM 2000, because all the vehicles that joined the queue within this analysis period were included in queue count until they crossed the stop line.

Cassette was played and the number of vehicles in queue was counted at regular interval of 10 to 20 seconds for analysis period of about 15 minutes. The regular interval was chosen in way that it was not an integral divisor of the cycle length of the studied intersection. Meanwhile it was ensured to keep track of end of standing queue by observing the last vehicle that in stops because of signal. This included vehicles arriving when the signal was actually green, but stopped

because vehicles in front did not yet started moving. The vehicles in queue counts often included some vehicles that regained speed, but did not yet exited the intersection. End of the survey period occurred when the last arriving vehicle (s) that stopped in the analysis period exited the intersection. Stopping vehicles that arrived after the end of the analysis period were not included in the final vehicle-in-queue counts.

The volume of total vehicles (V_{tot}) arrived during the survey period, and total vehicles arrived during the survey period that stopped one or more times were counted. A vehicle stopping multiple times was counted only once as a stopping vehicle (V_{stop}) as per HCM 2000 delay measurement guideline. Then the average time-in-queue delay per vehicle arriving in the survey period was estimated as:

Time-in-queue per vehicle, $$d_{vq} = 0.9\left(I \frac{\sum V_{iq}}{V_{tot}} \right) \tag{7}$$

Where,

I	=	Interval between vehicle-in-queue counts (sec)
V_{iq}	=	Sum of vehicle-in-queue counts (veh)
V_{tot}	=	Total number of vehicles arriving during the survey period (veh)

0.9 = an empirical adjustment factor accounts for the errors that may occur when this type of sampling technique is used to derive actual delay values, which normally results in an overestimation of delay (as per HCM 2000).

Next, the fraction of vehicles stopping and the average number of vehicles stopping in a queue in each cycle were computed.

Fraction of vehicles stopping, $$FVS = \frac{V_{stop}}{V_{stop}} \tag{8}$$

Finally, a correction factor (CF) given by HCM was selected based on average free flow speed (that was measured at the upstream of the selected approaches) and average number of vehicles stopping per queue in each cycle. The values of correction factor were obtained from Exhibit A16-2 of HCM manual (2000). The fraction of vehicles stopping was multiplied by the correction factor and the product was added to the time-in-queue delay value to obtain the final estimate of control (field) delay as shown below:

$$d = d_{vq} + d_{ad} \tag{9}$$

Where,

Acceleration-deceleration delay, $d_{ad} = FVS * CF$

Table 2 below shows the field measured delay values.

Table 2: Field Measured Delay

Intersection	Demand Flow Rate (veh/hr)	Saturation Flow Rate (veh/hr)	Capacity c=s(g/C) (veh/hr)	Degree of Saturation X=v/c	Time-in-Queue (Sec)	Acc/Dec Delay (Sec)	Control Delay (Sec)
New Market North Approach	940	3575	767	1.226	107.234	3.574	110.809
	1120	3575	767	1.460	117.000	3.786	120.786
	1152	3575	767	1.501	121.563	3.653	125.215
	1200	3575	767	1.564	133.980	3.733	137.713
	1160	3575	767	1.512	122.648	3.765	126.414
	1280	3575	767	1.668	148.668	1.950	150.619
Science Lab North Approach	1296	3029	1940	0.668	27.666	1.006	28.673
	1264	3029	1940	0.651	33.664	2.000	35.665
	1248	3029	1940	0.643	28.615	2.038	30.654
	1304	3029	1940	0.672	39.092	2.208	41.301
	1364	3029	1940	0.703	37.847	1.982	39.830
	1348	3029	1940	0.695	40.166	2.219	42.386
Science Lab East Approach	1104	3413	1263	0.874	41.673	2.812	44.486
	1048	3413	1263	0.830	49.534	2.855	52.389
	1052	3413	1263	0.833	54.889	3.118	58.008
Panthapath North Approach	988	4734	1171	0.844	62.162	1.514	63.676
	1000	4734	1171	0.854	74.664	1.616	76.280
	1224	4734	1171	1.045	93.353	1.804	95.157
	1164	4734	1171	0.994	85.299	1.759	87.058
	1116	4734	1171	0.953	89.032	1.770	90.803
Sheraton East Approach	1540	5257	2262	0.681	46.566	1.283	47.849

COMPARISON OF FIELD DELAY WITH THEORETICAL DELAY

This section compares the field measured delay values with the estimated delays using the five theoretical models as discussed in the section 5.0. Table 3 shows the values of field measured delay as well as the theoretical delays calculated by all those five models along with the respective relative errors. From the table it can be calculated that compared to field delay, the relative errors of Webster model varies from -121% to +5% with a standard deviation of 43.5, which is the minimum among all the models. However, it cannot estimate delay in case of oversaturated condition. So, this model is not suitable for intersection delay estimation for a city like Dhaka, where traffic at some intersections frequently faces oversaturated condition.

The maximum standard deviation of relative errors is associated with Akcelik model with relative errors ranging from -118% to +68% and a standard deviation of 63.6. TRANSYT 6 model also shows a poor performance with respect to standard error. The results of HCM 2000 and Reilly's models are found to be very close to each other, and they perform better than Akcelik and TRANSYT 6 models. The lowest and the highest standard errors for HCM 2000 and Reilly's models for the studied intersections are observed to be -98%, +65% and -118%, +68%, respectively. The corresponding standard deviations are pretty close, and these values differ at the second decimal point (56.11 and 56.10, respectively). The relative error has been calculated by the following formula:

$$\%RE = \frac{(MD - CD)}{MD} x100 \tag{10}$$

Where,

% RE = Relative error (%)

CD = Control (field) delay (sec/veh)

MD = Theoretical delay obtained from model (sec/veh).

Table 3: Values of Theoretical Delay and Field Measured Delay

Intersection	Theoretical Delay (sec/veh)					Control Delay (Sec/veh)	Relative Error				
	HCM 2000 Model	Akcelik Model	Reilly's Model	TRAN-SYT 6 Model	Webster Model		HCM 2000 Model	Akcelik Model	Reilly's Model	TRAN -SYT 6 Model	Webster Model
New Market North Approach	216.980	224.405	158.02	222.431	-	110.809	48.932	50.621	29.877	50.183	-
	342.025	358.257	228.89	355.036	-	120.786	64.685	66.285	47.229	65.979	-
	358.741	376.406	238.62	373.066	-	125.215	65.096	66.734	47.525	66.436	-
	391.371	411.224	257.05	407.734	-	137.713	64.813	66.511	46.425	66.225	-
	364.172	382.198	241.68	378.832	-	126.414	65.287	66.924	47.694	66.631	-
	445.944	469.572	288.01	465.893	-	150.619	66.225	67.924	47.704	67.671	-
Science Lab North Approach	20.682	18.839	18.84	20.682	18.749	28.673	-38.639	-52.199	-52.199	-38.639	-52.923
	20.212	18.497	18.50	20.212	18.486	35.665	-76.451	-92.812	-92.812	-76.451	-92.929
	19.986	18.331	18.33	19.986	18.356	30.654	-53.378	-67.224	-67.224	-53.378	-66.995
	20.803	18.926	18.93	20.803	18.817	41.301	-98.533	-118.222	-118.222	-98.533	-119.486
	21.769	19.608	19.61	21.769	19.338	39.830	-82.965	-103.131	-103.131	-82.965	-105.964
	21.502	19.422	19.42	21.501	19.196	42.386	-97.129	-118.236	-118.236	-97.129	-120.807
Science Lab East Approach	43.123	41.373	39.31	45.916	37.962	44.486	-3.159	-7.521	-13.169	3.115	-17.184
	40.082	38.455	37.41	42.808	35.535	52.389	-30.706	-36.234	-40.045	-22.382	-47.429
	40.267	38.632	37.53	42.998	35.661	58.008	-44.056	-50.156	-54.572	-34.906	-62.666
Panthapath North Approach	77.173	70.035	69.02	75.532	66.974	63.676	17.489	9.080	7.744	15.697	4.925
	77.915	70.691	69.46	76.268	67.588	76.280	2.098	-7.906	-9.822	-0.015	-12.859
	113.895	110.271	91.42	113.247	0.000	95.157	16.452	13.706	-4.082	15.975	-
	98.497	91.872	81.62	96.775	0.000	87.058	11.613	5.239	-6.669	10.041	-
	89.419	81.932	76.17	87.719	88.818	90.803	-1.547	-10.827	-19.207	-3.514	-2.235
Sheraton East Approach	47.063	36.253	36.25	37.929	34.912	47.849	-1.671	-31.987	-31.987	-26.153	-37.055

Figure 4 presents the graphical comparison of theoretical models with field measured delay. This figure shows two distinct features. When the degree of saturation exceeds 1.0, Reilly's model outperforms all other theoretical models considered in this study. However, HCM 2000 model gives better estimation of delay than the Reilly's one when the degree of saturation is less than 1.0. As except for the New Market North Approach, all other studied intersections show the degree of saturation values less than 1.0 for most of the time, HCM 2000 model seems to be better for the traffic situation in these studied intersections. Table 4 shows the root mean square errors (RMSE) and the R^2 values of the five theoretical models.

From Table 4, it is observed that Webster's model gives lower value of R^2 and higher value of RMSE. Besides, it has already been mentioned that Webster's equation cannot be applicable for *v/c* ratio greater than 1.0. Even this equation gives very high delay values for degree of saturation close to 1.0. Reilly's model and Akcelik's model give slightly higher value of RMSE and satisfactory value of R^2. These two models give very close results for isolated signalized intersection but do not take into account for the effect of signal coordination and uncoordinated nearby intersections. Both HCM 2000 delay model and TRANSYT-6 Model have satisfactory value of RMSE (less than 0.3) and R^2. Again among all of these models only HCM 2000 takes into account the effect of signal coordination and uncoordinated nearby intersections.

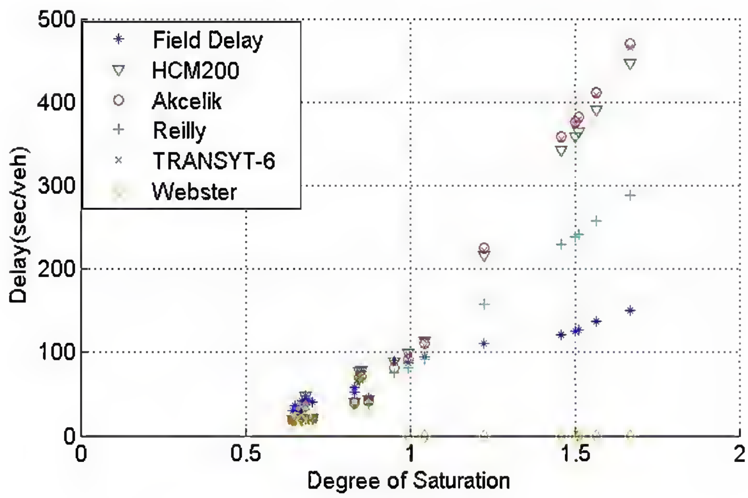


Figure 4: Comparison of Theoretical Delay with Field Measured Delay

Table 4: RMSE and R^2 Values between Observed Delay and Theoretical Delay Model

Delay Model	RMSE	R^2
HCM 2000 model	27.87213	0.926
Akcelik Model	32.18109	0.9279
Reilly's Model	34.34336	0.926
TRANSYT 6 Model	27.40745	0.9317
Webster Model	43.4365	0.9036

DELAY MODEL FOR NON-LANE BASED TRAFFIC CONDITION

It has been already mentioned that HCM 2000 delay model takes into account for the effect of signal coordination and uncoordinated surrounding intersections. Among all the theoretical models, HCM 2000 model is selected to modify so that it can estimate delay for non-lane based traffic condition more accurately. From the present study, it has been observed that HCM 2000 model consistently overestimates delay values at degree of saturation above 1.0 and underestimates delay values when it is less than 1.0. It has already been shown that theoretical delay of HCM 2000 model can be estimated by equation 6.

$$d = d_1 PF + d_2 + d_3 \qquad (6)$$

In this study, survey period was selected in such a way that there was no residual delay and hence d_3 is zero. For the purpose of regression analysis, above delay equation can be written as

$$d_f = c + ax_1 + bx_2 \qquad (11)$$

Where,

d_f = Field delay (sec/veh)

x_1 = $d_1 * PF$;

x_2 = $d_2/900$;

a and b = Calibration parameters;

c = Model intercept.

The target of this analysis is to find suitable values of the intercept (*c*), and the constant terms for d_1 (*a*) and d_2 (*b*). The current values of *a, b* and *c* are 1, 900 and 0, respectively. Value of saturation flow is one of the most important variables in finding out delay. For the purpose of regression analysis, average saturation flow of a particular approach that was observed over the entire survey period has been used (see Table 1). The modified delay estimation equation has been proposed based on regression analysis carried out by SPSS V11. Proposed modified HCM 2000 delay model along with goodness of fit statistics are given in table 5.

Table 5: Regression Results of Proposed Delay Model

Proposed Modified HCM 2000 Model : $d_f = 21.08 + 0.80\, x_1 + 132.20\, x_2$							
R^2	Co-Efficient			t-Value			F
	c	x_1	x_2	t_c	t_{x_1}	t_{x_2}	
0.967	21.08	0.80	132.20	5.54	9.76	7.84	287

The first term of the proposed model is the intercept which usually accounts for the effect of the variable (s) that might not been considered in this analysis. The second term is associated with the uniform delay and the third term is for the delay due to random arrival and oversaturation queues. The second term of the proposed model suggests that uniform delay equation gives lower estimate of field value to some extent, which is 0.80. The third constant is 132.20. In preparing models for the 1985 Highway Capacity Manual, Reilly et al. (1983) conducted extensive field studies to measure delay. They found that Akcelik's equation consistently overestimated field measured values, and recommended that the theoretical overflow delay results be reduced by 50% to better reflect field conditions. Present study establishes the similar fact. And from the proposed model, it is clear that overflow delay should be reduced by 85% and uniform delay should be decreased by 20% for non-lane based traffic condition.

The suggested delay equation shows good correlation with field measured delay. Obtained value of RMSE is 1.39 and R^2 = 0.967 for the suggested model which is shown in Table 5. Whereas those values were 27.87 and 0.926 respectively for HCM 2000 theoretical delay formula. Figure 5 presents the relationship between control (field) delays with the delay predicted by the modified HCM 2000 model. It can be observed that there is a very good agreement between these two delay values.

Figure 6 shows the prediction accuracy of HCM 2000 theoretical delay model and the proposed modified HCM 2000 model with respect to the control (field) delay values at all of the studied intersection approaches. In this study the measured control (field) delay is considered the ground truth. It can be clearly observed that the control (field) delay and the delay predicted by the proposed model are very close to each other with slight overestimation or underestimation for any degree of saturation (less than or greater than 1.0).

In Figure 6 the observation points from 1-6 and 18-20 are associated with degree of saturation values more than or very close to 1.0. In such cases the original HCM 2000 model consistently overestimates field delay. For all of the studied intersection approaches the absolute relative errors of the original HCM 2000 model vary from 1.52% to 196.07%, whereas the absolute relative errors of the proposed model vary from 1.23% to 26.93%.

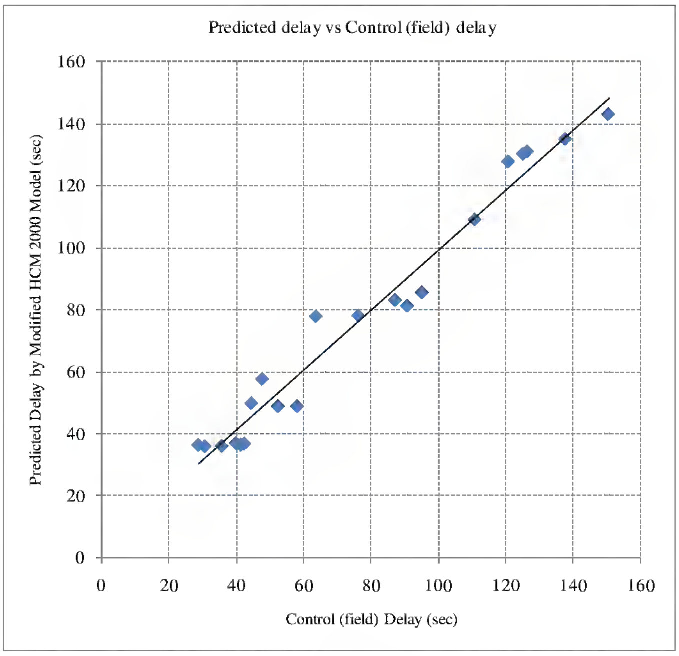


Figure 5: Correlation between Observed Delay and Suggested Delay Model

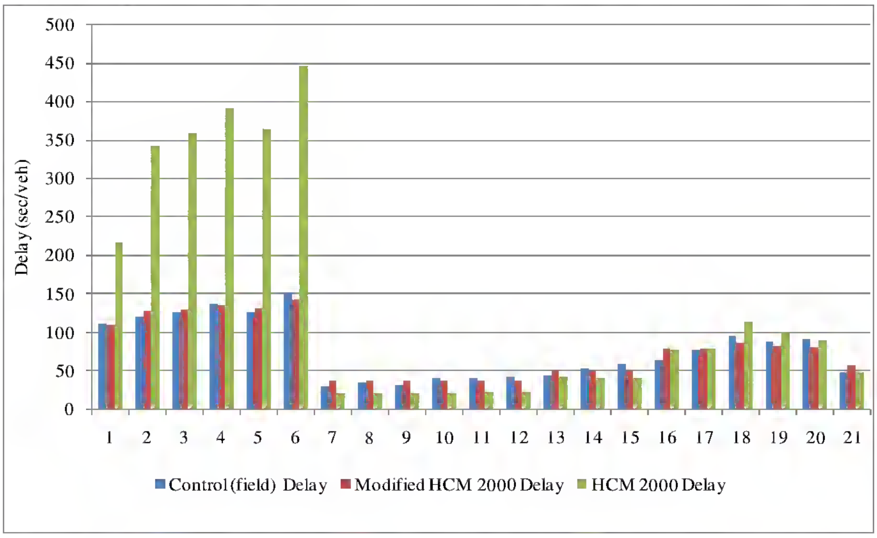


Figure 6: Comparison of Delay by Suggested Model with Field and HCM 2000 Delay

CONCLUSIONS

Delay is a very important parameter in capacity analysis of signalized intersections and measurement of LOS. HCM 2000 has defined six LOS based on control delay. This study gives insight into the field measurement of delay, theoretical estimation of delay and recommendation to HCM delay model to become applicable in non-lane based traffic condition especially for the city of Dhaka. The proposed model concludes that both of the overflow delay and the uniform delay of the HCM 2000 delay model should be reduced by 20% and 85%, respectively for non-lane based traffic condition. The proposed delay model shows good correlation with field measured delay with a R^2 value of 0.967. However, this model proposed an intercept term to capture the effect of other variable (s) that might be related to non-lane based traffic flow but yet not has been included in this model. The study revealed that the control (field) delay and the delay predicted by the proposed model are in a good agreement to each other with slight overestimation or underestimation. The most

interesting feature of the proposed model is that it can predict field delay both for the undersaturated and oversaturated flow condition with sufficiently high accuracy.

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